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PINOLE SLIDE
I-80, CALIFORNIA

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Introduction

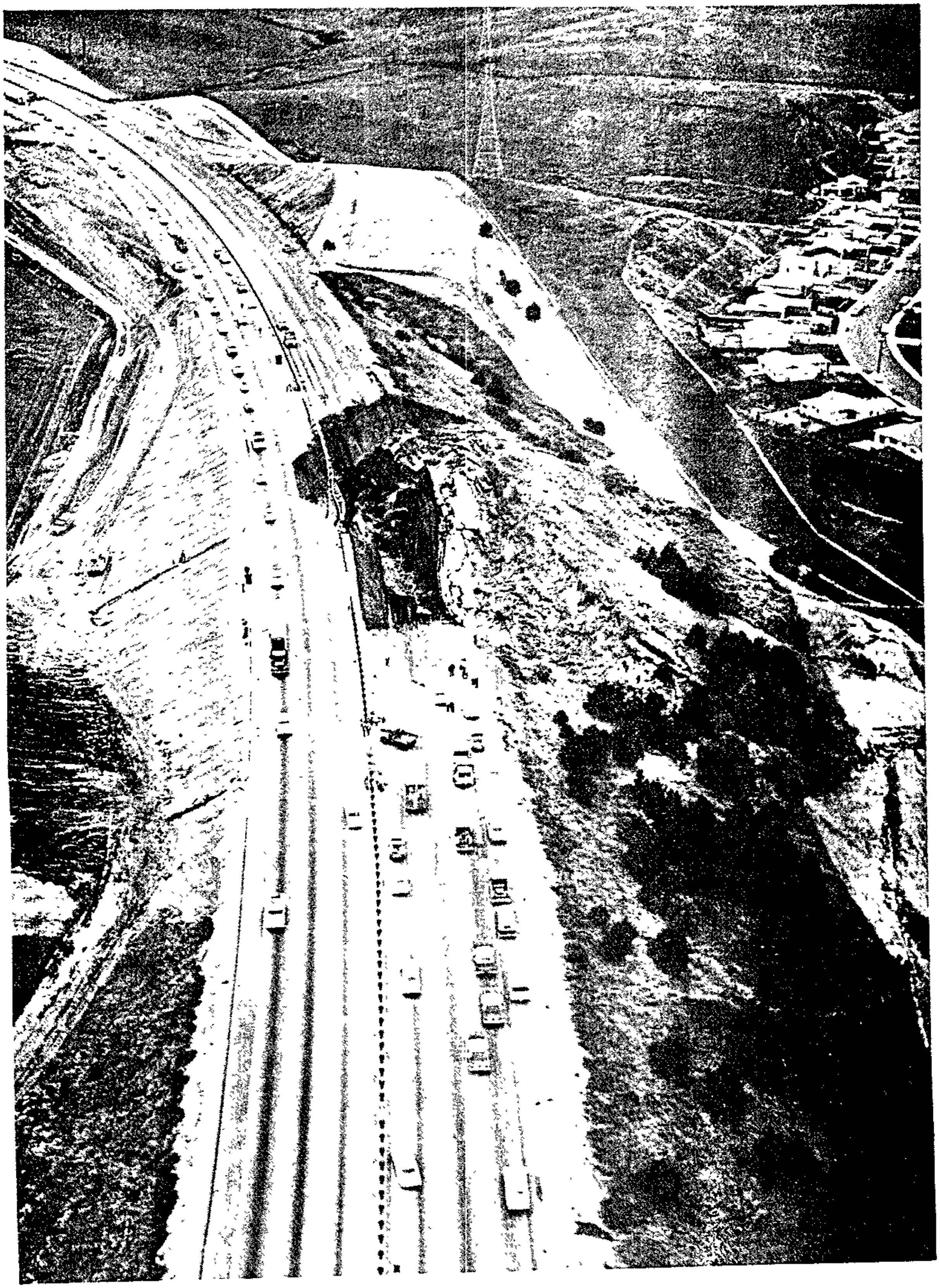
On Tuesday, May 6, 1969, indications of distress in the form of cracking and settlement appeared on a portion of the pavement of Interstate Route 80 near Pinole, California. Efforts to restore the damaged surface of the heavily traveled freeway during the next few days were ineffective as the deterioration of the roadway progressively worsened. It soon became apparent that the distress reflected an unstable condition within the embankment or its underlying foundation, and by Sunday morning, May 11, the prospect of a massive failure appeared imminent. It was therefore decided to close the road at midday in the interest of public safety. The Highway Patrol was notified, traffic was diverted, and police forces in nearby cities were alerted to prepare for unusual traffic over alternate routes. A few hours after these precautions were taken, over 400 feet of embankment slid out leaving a single lane of the roadway in place, as shown in Photographs 1 and 2.

This paper describes the ensuing investigation of this abrupt and total disruption of a major arterial highway and the corrective treatment employed to restore the roadway to a permanent and serviceable condition.

Background

The construction of the portion of what is now Interstate Route 80 between the city of San Pablo and the Carquinez Bridge was completed in 1958. This route is the principal highway for the east-bay area north of El Cerrito and forms part of the main transcontinental system providing all year connection with areas east of the Sierra Nevada mountains. Location of the freeway is shown in Figure 1.

The terrain in the vicinity of the slide is comprised of rolling, grassy hills with sparse rocky outcrops. The embankment that failed crossed a natural northerly bearing drainage swale in a northeasterly direction.



Photograph No. 1 - Aerial photo of slide, looking westerly

Photograph No. 2 - Aerial photo of slide, viewed from lower side.

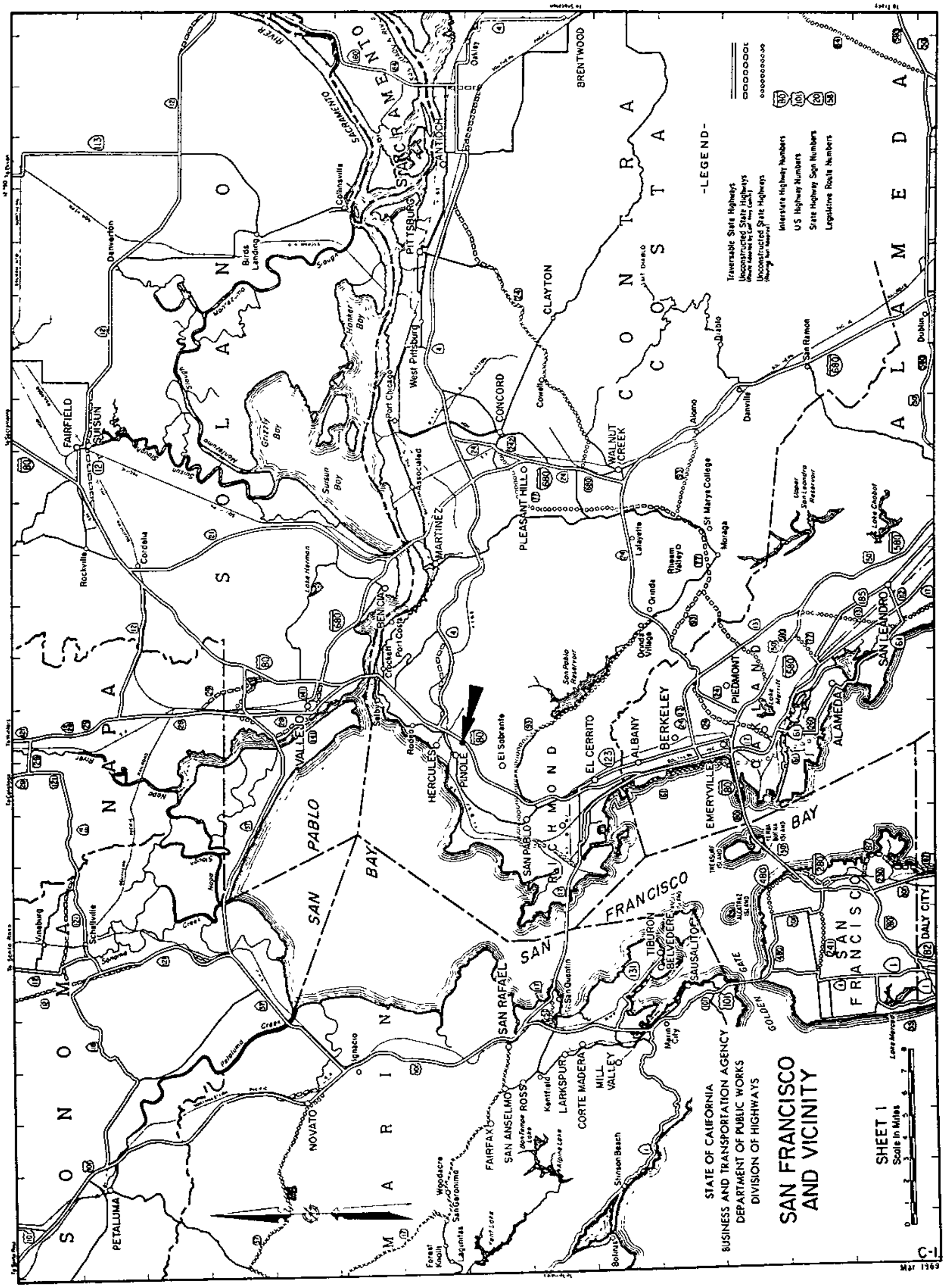


Figure 1 - Map of area with location of embankment failure indicated

The roadway is a 6-lane portland cement concrete paved divided facility with a 12-foot median and 8-foot shoulders. Maximum embankment heights in the failed area were 60 feet at roadway centerline, 84 feet at the north toe and 46 feet at the south toe. The original embankment was 106 feet wide at roadbed level with 2:1 side slopes.

A preliminary field exploration for evaluation of the foundation soils had been conducted early in 1954. Ninety 1-inch soil sample borings were made in nine proposed fill areas using the California soil sampler to provide information on soil classification, density, moisture content, and resistance to penetration. In those areas where potential weakness was indicated by the borings the data was augmented by deeper power borings.

The 1-inch borings taken from proposed fill areas revealed a general pattern of two to three feet of black, plastic clay over brown silty and sandy clays. Those borings made in existing drainage courses generally indicated wet, plastic surface clays to varying depths which would require removal. Water was seldom encountered along the alinement and in most locations no need of special treatment was indicated.

While the original exploration conducted in 1954 was quite extensive for the project as a whole, the limited investigation made in the immediate area of the failure did not reveal any special problems. Competent foundation materials were noted in the two 4' to 6' deep borings that were taken and ground water was not encountered. The recommendation included in the Materials Report for this location involved removal of 3 feet of black surface clay from the foundation area of a culvert scheduled for the drainage course.

Reconstruction of the failed embankment embodied three general stages. Initially, the work involved construction of a detour to carry the extremely large traffic volumes; secondly, an investigation and analysis of the causes of the failure; and, finally, permanent reconstruction of the embankment and roadway incorporating corrective measures determined as a result of the investigation.

Detour Construction

The closure of the freeway caused great inconvenience to the public, especially the many thousands of commuters of the east-bay area who were forced to revert to roundabout alternate routes that were abandoned as major commuter and through roads a decade earlier when the freeway was constructed.

It was imperative that a local detour be constructed around the damaged area without delay and the Division of Highways moved swiftly to this end. The Director of Public Works approved arrangements for a major construction firm, working on a nearby project, to move in heavy equipment and commence work on a detour under an emergency force account contract. Operations started on this phase the same day that the slide occurred.

The detour, approximately 1/3 mile long, provided six 12-foot lanes, a 12-foot median and 8-foot shoulders, and involved the excavation and placement of 100,000 cubic yards of material. Seventy-five trucks were used to haul 12,000 tons of asphalt and 20,000 tons of aggregate base material. A force of 400 men was employed on the job for both Contractor and the Division of Highways, with alternate crews working on a 24-hour basis.

This $350,000 detour was opened to full traffic service within a week of the slide and served to handle freeway traffic flow while the failure was investigated and during the permanent reconstruction. Location of the detour is shown on Figure 2.

Failure Investigation

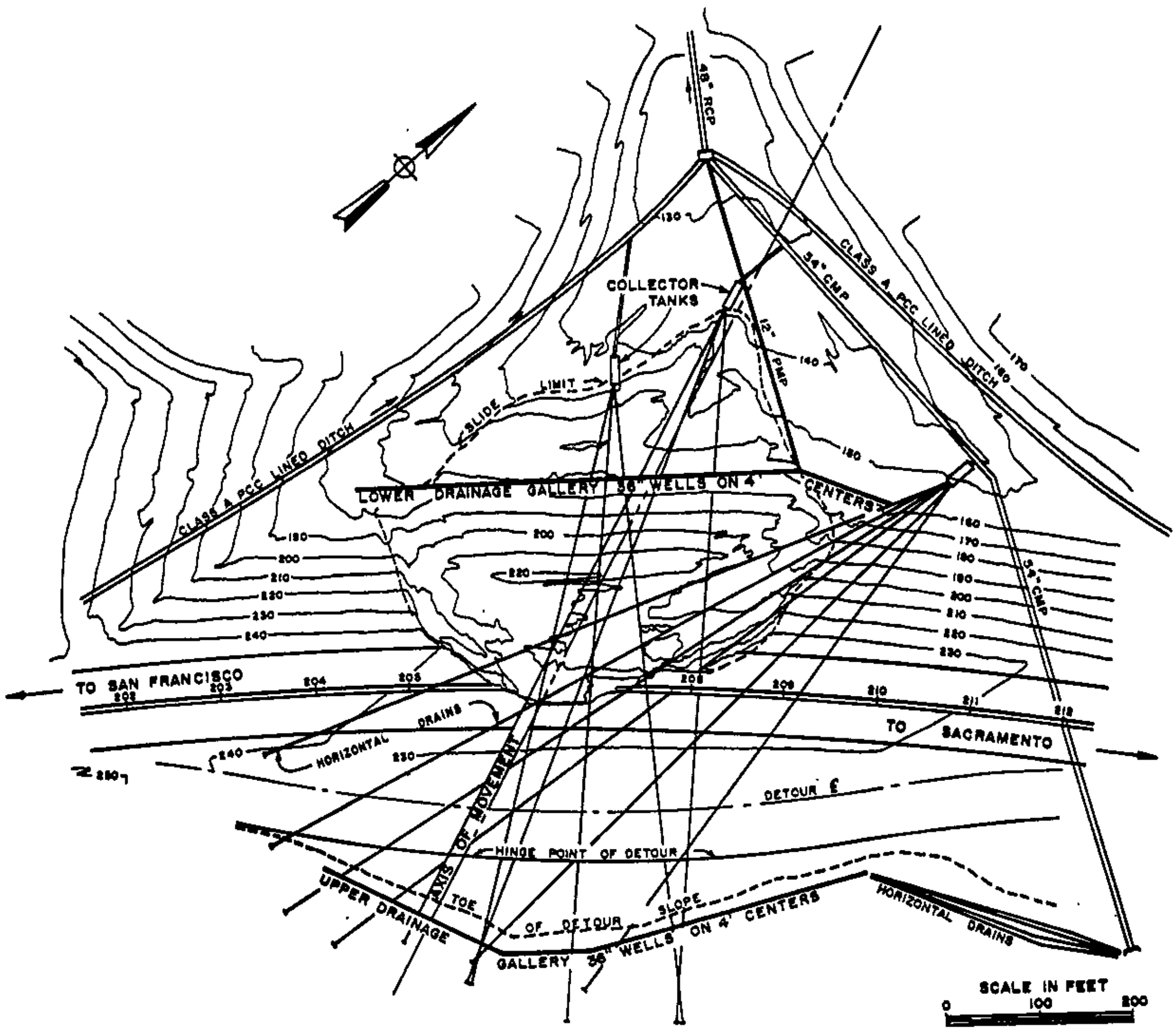
The investigation of the failure began with the immediate dispatch of drill rigs and crews to the scene. Four crews using varied equipment worked alternate shifts round the clock making borings and removing soil samples from the affected areas for laboratory evaluation and analysis. Thirty-six vertical borings were completed for a total of 1,622 feet of vertical hole. Inclinometer installations were made in nine of the boreholes totaling 660 feet in length to determine depths of subsurface movement.

It became evident within a few days that ground water in fairly large quantity was present under the failed area. Ground water encountered at depths of 20 to 25 feet below ground surface near the toe of the detour embankment rose 10 to 15 feet in the bore holes. Ground water was also observed near the toe of the slide under a slight head. These observations indicated that excessive amounts of seepage water had become entrapped beneath the embankment and that the resultant hydrostatic pressures had triggered the failure in the embankment foundation.

Steps taken for the immediate relief of the impounded water included pumping from two vertical wells placed "upstream" at the detour embankment toe and the installation of a comprehensive system of horizontal drains. The wells were dewatered six times a day to a depth of 33 feet, producing a total of 1,400 gallons per day.

Twelve horizontal drains were installed in fan patterns from three locations at two different elevations. These drains, varying between 550 and 830 feet in length, produced a total flow of 10,000 to 12,000 gallons per day, the largest flows having been produced by a group of three drains drilled into the center of the failed area. Location of horizontal drains is shown in Figure 2. During the six-week period following their installation, these drains lowered ground water 7 feet at the toe of the detour, 2.3 feet beneath the central portion of the slide mass, and 1 foot at the toe of the slide.

A laboratory testing program was begun upon receipt of the first soil samples from the field. The major portion of the program, which included



PLAN OF FAILED AREA
STA. 202 TO STA. 212

Figure 2 - Plan of failure area showing drainage system and detour

routine classification tests, was devoted to determining soil strength by triaxial compression tests. These tests, scheduled daily, were based on a continuing review of boring logs and samples as they were received. Most of the tests were conducted on unconsolidated-undrained specimens although several consolidated-undrained, consolidated-drained and unconfined compression tests were included in the program. The unconsolidated-undrained tests were about equally divided between laboratory saturated specimens (with pore pressure measurement) and specimens at field moisture content. Because of time limitations, testing was restricted primarily to samples from six borings nearest the axis of mass movement.

An examination of data from the field exploration and laboratory testing programs showed a complex, nonuniform, subsurface stratification consisting of firm to stiff silty clays, sands with small amounts of gravel, and sandy clays. Predominant materials consisted of sandy silty clays with the three fractions being present in more or less equal percentages.

With the exception of the sands and a few silty clay samples bordering the sands, the soils were only damp to slightly moist and showed no evidence of free or excess water despite the fact free water was encountered in practically all borings. This is undoubtedly the result of the impermeable nature of the clays which created an artesian condition in the sandy aquifer. The lack of excess water was also reflected in triaxial compression tests on specimens at field moisture contents, compressive strengths of which generally ranged between 2 and 6 TSF. The same materials, when saturated and tested under undrained conditions, exhibited compressive strengths in the order of 0.7 to 1.5 TSF. It is therefore considered that failure resulted from a strength loss due to excess hydrostatic pressure within the saturated silty clay bordering the sandier deposits that functioned as aquifers. The depths at which free water was encountered varied from 20 to 35 feet between borings but the most frequent depths were 23 to 28 feet below original ground surface. Almost without exception, water rose in the borings.

A stability analysis was performed by computer on a section through the axis of movement. Assumptions included a circular failure surface and $\phi = o$ conditions for soil strength. Since the foundation soils in the failure zone were saturated and in an undrained condition the $\phi = o$ analysis was considered appropriate. Strength values representative of saturated unconsolidated undrained conditions were selected from test data. Values of cohesion ranged from 0.25 tons per square foot for the black clay layer at the surface of the soil profile to 1.00 TSF for the clayey sandy silt some 23 feet below the original ground surface. Seepage forces were not introduced in the analysis because of the apparent low flow gradient which would result in very small increases in driving forces.

Three trial circles were used as starting points in a computer search program in which the radius is first varied to determine a minimum safety factor and then the circle center is varied to find another minimum safety factor. For the conditions analyzed, a minimum safety factor of 1.01 was obtained for the circle shown on Figure 3.

The mode of failure appears to have been more of the translational type than circular, especially through the lower area of movement. The use of a circular failure surface as an approximation is believed to have been justified since the critical circle was fairly flat through the zone of original ground and conformed reasonably well to the available inclinometer data. Also the safety factor of unity attested to the reasonableness of strength values used in the analysis.

Stability computations were also performed to determine the effects of possible corrective treatment. One such treatment consisted of constructing a berm parallel to and downhill of the existing embankment to provide a counterweight against potential future movement. The possibility of constructing a stabilization trench within the slide zone, was also considered. Since this construction would require removing a portion of the slide material to a depth of 30 to 40 feet below original ground, a condition of questionable stability would be temporarily created. This condition was analyzed, found to be marginal, and not considered further.

Conclusions From Failure Investigation

Evaluation of data based on visual observations, borings, field measurements and laboratory tests led to the conclusion that the failure resulted from a loss in strength of foundation soils due to an unprecedented rise in the water table. This rise followed a severe series of winter rainstorms that exceeded seasonal normals for the area both in terms of total precipitation and number of rain days.

A second factor believed contributory to the sharp rise of the water table was the construction in 1964 of two 25 foot high fills within a subdivision developed "downstream" near the west right of way line about 8 years after completion of the highway embankment. Erected astride the natural drainage channel, these fills presumably constricted or pinched off subsurface drainage aquifers, aggravating the increase in water elevation.

Inspection of the sheared face of the highway embankment indicated that it had been of sound construction. It was dry, well compacted, and the material was of good quality.

Corrective Measures

Prior to reconstruction of the freeway a comprehensive drainage system was installed to safeguard the embankment under the worst weather conditions. Gravel-filled drainage galleries 600 feet in length were installed parallel to the toe of each slope. These galleries consisted of rows of augered holes, 60 to 70 feet in depth, 36 inches in diameter, and spaced on 4-foot centers. These holes were belled to connect at their bases to form a continuous gallery beneath the ground surface. Water intercepted by the

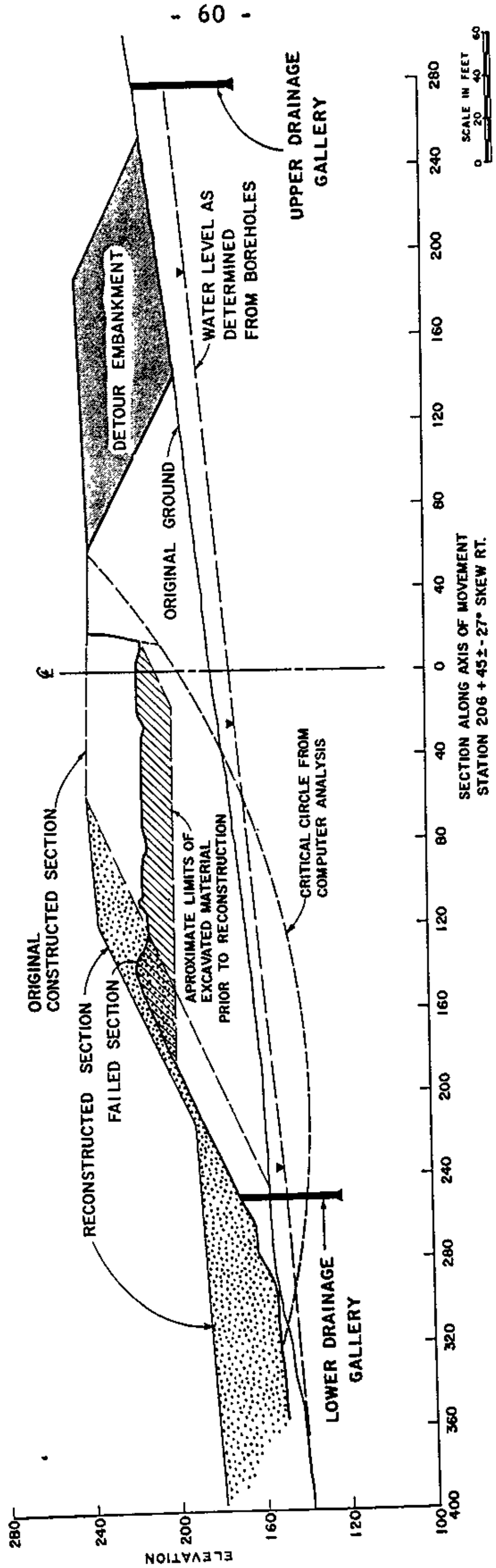


Figure 3 - Cross section along axis of movement, Station 206+45 ±, showing failure plane, reconstruction features and detour embankment

gallery on the high side of the fill is drained laterally to a collecting basin at ground surface by means of a 6-inch steel pipe supplemented by a group of three short horizontal drains. From this point the cumulative flow is conducted beneath the embankment to the drainage disposal area through a 54-inch by 990-foot CMP. A 12-inch by 350-foot perforated metal pipe laid in an excavated trench bedded with gravel drains the lower gallery to the master drain junction box (see Figure 2).

The 12 horizontal drains installed to relieve ground water pressure were incorporated into the permanent drainage system. Three 32-foot lengths of 120-inch multiplate pipe fitted with concrete ends to form tanks were utilized to collect flows from the three groups of drains. Water collected in the tanks exited through small conduits into the large 54-inch pipe drain and into the open PCC lined ditches that function as surface drains along the toes of the embankments (see Figure 2).

Increased stability of the embankment was provided by the placement of a 25-foot high berm downhill of the roadway, extending laterally 50 feet beyond the downhill limit of movement (see Figure 3). This berm was keyed into the existing subdivision embankments northerly of the original embankment toe. A factor of safety in the order of 1.4 was obtained through employment of the berm.

Reconstruction of the failed embankment took place during the period from August 1969 to April 1970. The total cost was approximately 1.25 million dollars.